Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability

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SUMMARY-1

Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

Practical simplified seismic stability analysis needs appropriate balance among the methods chosen in the following items:

1) Criterion to evaluate of the stability:
   Global safety factor relative to a specified required minimum vs. Residual deformation relative to a specified allowable largest.

2) Design seismic load at a given site:
   Conventional design load vs. Likely largest load in the future

3) Stress – strain properties of soil:
   Actual complicated behavior vs. Simplified model

4) Relevant consideration of the effects of other engineering factors:
   - compacted dry density; soil type; etc.
SUMMARY-2

Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

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2) Design seismic load at a given site:
   - Conventional design load vs. Likely largest load in the future

3) Stress – strain properties of soil: actual complicated behavior vs. Simplified model

4) Relevant consideration of the effects of other engineering factors:
   - compacted dry density; soil type; etc.
Stress – strain properties of soil:

(A) actual complicated behaviour vs. (B) simplified model

(A) actual complicated behaviour
a) Peak strength corresponding to actual compacted dry density
b) Anisotropic stress – strain properties as a function of $\delta$
c) Plane strain condition in many cases
d) Strain-softening associated with shear banding with the thickness increasing with $D_{50}$
e) Progressive failure as a result of d) among others.

(B) Constant strength irrespective of $\varepsilon$

- Peak strength only at a certain $\varepsilon$
- Strain-softening
- Residual strength
- Constant strength irrespective of $\varepsilon$
- Shear band
- Plane strain conditions
Stress – strain properties of soil:

(A) actual complicated behaviour vs. (B) simplified model

(B) simplified model (explained in this presentation)

a) Design strength corresponding to conservatively (but not excessively) determined compacted dry density
b) Isotropic stress – strain properties
c) Strength by triaxial compression test at $\delta = 90^\circ$
d) Strain-softening associated with shear banding with the thickness increasing $D_{50}$ to account for the effects of compaction & particle size
e) No progressive failure in the limit equilibrium-based stability analysis

Good balance is required among simplifications a), b), c) and e) d) is to encourage good compaction

Discussions on these topics a) - e)
Conservative determination of design soil shear strength under drained conditions - 1

The degree of compaction

\[ D_c = \frac{\rho_d \text{ (in-situ)}}{(\rho_d)_{\text{max}} \text{ (laboratory tests)}} \times 100 \text{ (%)} \]

Average of actual shear strength

\[ \rho_d \text{ (in-situ)} \] in actual construction:

- average each measurement

Allowable lower bound of \( \rho_d \) (e.g., \( D_c = 90 \% \)) in field compaction control

Design shear strength - Often employed, but too conservative when well-compacted

Laboratory compaction curve by specified compaction energy level (CEL)

\[ \text{Dry density, } \rho_d \]

\[ \text{Water content, } w \]

Zero air voids (\( S_r = 100 \% \))

Average of actual shear strength

\[ \rho_d \text{ (in-situ)} \] in actual construction:

- average each measurement

Allowable lower bound of \( \rho_d \) (e.g., \( D_c = 90 \% \)) in field compaction control

Design shear strength - Often employed, but too conservative when well-compacted

Laboratory compaction curve by specified compaction energy level (CEL)
Design shear strength is often determined to correspond to the allowable lower bound of $D_c$ used in field compaction control $\Rightarrow$ conservative with better compacted soil.
Conservative determination of design soil shear strength under drained conditions- 2

Drained TC at $\sigma_3' = 50$ kPa

$$\phi_{\text{peak}} = \arcsin\left[\frac{\sigma_1' - \sigma_3'}{\left(\sigma_1' + \sigma_3'\right)}\right]_{\text{max}}$$

The use of the design peak shear strength that is slightly lower than the value that corresponds to the target of $D_c$ set equal to the anticipated average of actual values, together with the residual shear strength, is more realistic and can encourage better compaction (explained later). 

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Inherently anisotropic stress-strain behaviour under drained conditions - 1

Pluviation through air

Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability

Drained PSC, saturated Toyoura sand

Principal stress ratio, \( \sigma'_1/\sigma'_3 \)

Volumetric strain, \( \varepsilon_{vol} \)

Axial strain, \( \varepsilon_1 \) (%)
Inherently anisotropic stress – strain behaviour under drained conditions - 2

**Summary**

![Graphs showing stress-strain behaviour](image)

**Dense**

\[ \varepsilon_{4.9} = 0.685 - 0.714 \] (except **a** with 0.666)

<table>
<thead>
<tr>
<th>( \sigma'_3 ) (kPa)</th>
<th>Average (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.9</td>
<td>9.8</td>
</tr>
<tr>
<td>49</td>
<td>98</td>
</tr>
<tr>
<td>392</td>
<td></td>
</tr>
</tbody>
</table>

**Loose**

\[ \varepsilon_{4.9} = 0.770 - 0.805 \] except **b** (0.839) & **c** (0.836)

<table>
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<td></td>
</tr>
</tbody>
</table>

**Average for loose specimens**

**Drained PSC, saturated Toyoura sand**

- **\( \sigma'_3 = 392 \text{ kPa, dense} \)**
- **\( \sigma'_3 = 392 \text{ kPa, loose} \)**

**Principal stress ratio, \( \sigma'_1/\sigma'_3 \)**

**Volumetric strain, \( \varepsilon_{\text{vol}} \)**

**Axial strain, \( \varepsilon_1 \) (%)**
Inherently anisotropic stress – strain behaviour under drained conditions - 3

A similar trend among different poorly-graded sands collected from different countries, with and without a minimum at $\delta = 20^\circ - 30^\circ$, where the shear band direction coincides with the bedding plane direction.
Inherently anisotropic stress – strain behaviour under drained conditions - 4

A similar trend among different poorly-graded gravelly soils produced by vertical vibratory compaction
Inherently anisotropic stress – strain behaviour under drained conditions - 5

The TC strength (δ = 90°) is similar to, or smaller than, the average strength along a circular failure plane under plane strain conditions.
Inherently anisotropic stress – strain behaviour under drained conditions

Air-pluviated Toyoura sand

The TC strength (δ= 90°) is noticeably smaller than the average strength along a circular failure plane under plane strain conditions.
Inherently anisotropic stress – strain behaviour under drained conditions: 7

\[ \phi_0 = \arcsin\left[\frac{(\sigma'_1 - \sigma'_3)}{(\sigma'_1 + \sigma'_3)}\right] \]

values in the direct shear test and the PSC test (\(\delta = 40^\circ-50^\circ\)) are nearly the same, because both tests are plane strain tests with similar anisotropy effects.

\[ \phi_{ss} = \arctan\left(\frac{\sin \phi_0 \cdot \cos(\nu_d)_{\text{max}}}{1 - \sin \phi_0 \cdot \sin(\nu_d)_{\text{max}}}\right) \]

Stress condition in TSS test

\[ \sigma'_3 \quad \tau_d \quad \sigma'_a \quad \sigma'_1 \]

\[ \epsilon_2 = 0 \quad \sigma'_2 \quad \epsilon_n = 0 \]

Air-pluviated Toyoura sand

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Inherently anisotropic stress – strain behaviour under drained conditions

\[ \phi_0 = \arcsin\left(\frac{(\sigma'_1 - \sigma'_3)}{(\sigma'_1 + \sigma'_3)}\right) \]

values in the direct shear test and the TC test \((\delta = 90^\circ)\) happen to be nearly the same due to cancelling out of the effects of anisotropy and \((\sigma'_2 - \sigma'_3)/(\sigma'_1 - \sigma'_3)\).

\[ \phi_{ss} = \arctan\left(\frac{\sin \phi_0 \cdot \cos(\nu_d)_{\max}}{1 - \sin \phi_0 \cdot \sin(\nu_d)_{\max}}\right) \]

Stress condition in TSS test

\[ \sigma'_1, \sigma'_2, \sigma'_3, \tau_{at}, \sigma'_a \]

Measured \( \phi_{ss} = \arctan(\tau_{at}/\sigma'_a) \) (TSS, \( \sigma'_a = 98\, kPa \))

when \( \tau_{at}/\sigma'_a = \text{max} \)

when \( \sigma'_1/\sigma'_3 = \text{max} \)
In ordinary direct shear tests, $\phi_0 = \arcsin\left[\frac{(\sigma'_1 - \sigma'_3)}{[(\sigma'_1+\sigma'_3)]_{\text{max}}}\right]$ cannot be measured, but only $\phi_{ss} = \arctan\left(\frac{\tau_{at}}{\sigma'_a}\right)_{\text{max}}$ is measured.

Inherently anisotropic stress – strain behaviour under drained conditions

Air-pluviated Toyoura sand

Stress condition in TSS test

Theoretical value

$\phi_{ss} = \arctan\left(\frac{\sin\phi_0 \cdot \cos(\nu'_d)_{\text{max}}}{1 - \sin\phi_0 \cdot \sin(\nu'_d)_{\text{max}}}\right)$

Measured $\phi_{ss} = \arctan\left(\frac{\tau_{at}}{\sigma'_a}\right)$ (TSS, $\sigma'_a = 98\,\text{kPa}$) when $\tau_{at}/\sigma'_a = \text{max}$

when $\sigma'_1/\sigma'_3 = \text{max}$

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17
Inherently anisotropic stress – strain behaviour under drained conditions - 10

φ_{ss} = \arctan\left(\frac{\tau_{at}}{\sigma'_a}\right)_{\text{max}}\) from the direct shear test is significantly lower than \(\phi_0\) from TC test (δ = 90°).

The use of \(\phi_{ss}\) in the slope stability analysis is usually too conservative.

\[ \phi_{ss} = \arctan\left(\frac{\sin \phi_0 \cdot \cos (\nu_d)}{1 - \sin \phi_0 \cdot \sin (\nu_d)}\right)_{\text{max}} \]

Stress condition in TSS test

\begin{align*}
\sigma'_3 & \quad \tau_{at} \quad \sigma'_a \\
\sigma'_1 & \quad -\tau_{at} \quad \sigma'_2 \\
\sigma'_2 & \\
\epsilon_2 & = 0 \\
\epsilon_n & = 0
\end{align*}

Air-pluviated Toyoura sand

Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability | 2016 18


Inherently anisotropic stress-strain behaviour under drained conditions - 10

\[ \phi_0(\delta) \text{ when } e_{4.9} = 0.7 \]

\[ \phi_0(\delta) \text{ when } e_{4.9} = 0.8 \]

\[ \phi_{ss} = \arctan\left( \frac{\tau_{at}}{\sigma'_{a}} \right)_{\text{max}} \] from the direct shear test, much lower than the average strength along a circular failure plane under plane strain conditions.
Strain-softening associated with shear banding under drained plane strain conditions - 1

The stress is plotted against “strain averaged for the whole specimen”, not representative of the strain in the shear band.
Strain-softening associated with shear banding under drained plane strain conditions

$u_s$: shear displacement along a shear band

$\left(u_s\right)_{\text{peak}}$: values of $u_s$ at the peak stress

(very small)
Strain-softening associated with shear banding under drained plane strain conditions - 3

PSC tests on many poorly- & well-graded gravelly soils

Andesite 2

\(D_{50} = 2.49 \text{ mm} \quad \text{and} \quad U_c = 4.1\),

at \(\varepsilon_1 = 4.25\%\), \(\sigma'_3 = 314 \text{ kPa}\)
Strain-softening associated with shear banding under drained plane strain conditions - 3

PSC tests on many poorly- & well-graded gravelly soils

Larger shear deformation of shear band for larger $D_{50}$

Poorly graded granular materials

(Yoshida and Tatsuoka 1997)

Larger $D_{50}$

$(u_s^*)_{res}$

0.05
Strain-softening associated with shear banding under drained plane strain conditions - 5

$(u_s^*)_{res}$ increases with shear band thickness, $t_r$, for a similar shear strain in the shear band

At the start of residual state

Residual shear displacement, $(u_s^*)_{res}$

Thickness of shear band, $t_r$ (mm)

Linear fitting:
$y = 0.7989x$
$R^2 = 0.7288$

Non-linear fitting
$y = 1.44x^{0.76}$
$R^2 = 0.803$

Poorly graded
(Yoshida and Tatsuoka 1997)

Well-graded
(Okuyama et al. 2003)

$t_r$ increases with $D_{50}$

Shear band thickness, $t_r$ (mm)

Regression curve
$y = ax^{0.66}$
$R^2 = 0.89$

Slightly non-linear relation

Particle size, $D_{50}$ (mm)

Particle mean diameter, $D_{50}$ (mm)

So, $(u_s^*)_{res}$ increases with $D_{50}$

(Yoshida & Tatsuoka (1997)
Okuyama et al. (2003)
Average)
Strain-softening associated with shear banding under drained plane strain conditions - 6

Poorly- & well-graded gravelly soil

Normalization as $u_s/(D_{50})^{0.66}$: still a noticeable scatter, but no systematic effects of $U_c$, $\sigma_3$, density, strain rate, ..... Useful to infer the $R_n - u_s$ relation for a given $D_{50}$ to be used in the slip displacement analysis by the Newmark method.
On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the $D_{\text{max}}$/specimen size constant! This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.
On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the $D_{\text{max}}$/specimen size constant! This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.
Strain-softening associated with shear banding under drained plane strain conditions- 9

On the other hand, the friction angle decreases with an increase in the particle size in drained TC keeping the $D_{\text{max}}$/specimen size constant! This trend is inconsistent with our intuition that the slope becomes more stable with an increase in the particle size.

One method to alleviate this contradiction in the seismic design, at least partly, is the evaluation of slip displacement by the Newmark method taking into account the effects of $D_{50}$ on the $R_n - u_s$ relation.
Undrained stress-strain behaviour of saturated soil

1. Effects of dry density:
   - much larger than those on drained strength; and
   - become more significant by effects of preceding cyclic undrained loading.

2. Degradation of the undrained stress-strain properties and strength in the course of cyclic undrained loading:
   - more when more cyclically sheared undrained; and
   - how to model this trend for numerical analysis?

Simplified model for simplified numerical analysis* vs. Full model for rigorous numerical analysis

(* explained in this presentation)
Undrained stress-strain behaviour of saturated soil:

1. Effects of dry density - 1

Torsional simple shear
Toyoura sand
Consolidated at
\( \sigma'_{vc} = \sigma'_{vc} = 98 \, \text{kPa} \)

The figures indicate \( D_r (\%) \).

Range of drained peak strength state for \( D_r = 33.6 \% - 94.4 \% \)

Drained

Undrained
Undrained stress-strain behaviour of saturated soil:
1. Effects of dry density - 2

In drained tests, the peak strength is noticeably different with largely different volume changes for different dry densities (or different $D_r$ values)!
Undrained stress-strain behaviour of saturated soil:
1. Effects of dry density - 3

In undrained tests, the effective stress path is largely different with largely different peak strengths for different densities!
Undrained stress-strain behaviour of saturated soil:

1. Effects of dry density - 4

Comparison among
(A) drained strength in ML,
(B) undrained strength in ML and
(C) undrained CL strength of Toyoura sand

Shear strength, \( \tau_{vh} / \sigma'_v \)

Relative density, \( D_r \) (%)

Undrained cyclic test

A: drained peak stress ratio

B: undrained peak stress ratio necessary to develop 15% shear strain

C: undrained cyclic loading strength necessary to develop 15% double amplitude shear strain

Toyoura sand; \( \sigma'_{vc} = \sigma'_{vc} = 98 \text{ kPa} \)
### Undrained stress-strain behaviour of saturated soil:

#### 1. Effects of dry density - 5

![Graph showing stress-strain behaviour](image)

<table>
<thead>
<tr>
<th>Number of loading cycle, $N_c = 5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative density, $D_r$ (%)</td>
</tr>
</tbody>
</table>

- **A:** Drained peak stress ratio
- **B:** Undrained peak stress ratio necessary to develop 15% shear strain
- **C:** Undrained cyclic loading strength necessary to develop 15% double amplitude shear strain

1) ML drained shear strength increases with $D_r$, but the increase is not very large: e.g., only about 10% when $D_r = 70 \% \rightarrow 90 \%$.

2) In the stability analysis based on the drained shear strength, the benefit of compaction is large, but not as large as the one when based on undrained shear strength.
Undrained stress-strain behaviour of saturated soil:
1. Effects of dry density - 6

1) ML undrained shear strength significantly increases with $D_r$: e.g., by a factor of three when $D_r = 40\% \rightarrow 60\%$.

2) In the stability analysis based on the undrained shear strength, the benefit of compaction is significant.

A: drained peak stress ratio

B: undrained peak stress ratio necessary to develop 15 % shear strain

C: undrained cyclic loading strength necessary to develop 15 % double amplitude shear strain

Toyoura sand; $\sigma'_{vc} = \sigma'_{vc} = 98$ kPa
Undrained stress-strain behaviour of saturated soil:

1. Effects of dry density - 7

1) Undrained cyclic shear strength increases with $D_r$, significantly when $D_r$ becomes larger than a certain value: e.g., by a factor of three when $D_r = 70\% \rightarrow 90\%$.

2) Significant benefits can be obtained by compaction to $D_r$ higher than a certain value.
Undrained stress-strain behaviour of saturated soil:
1. Effects of dry density - 8

These undrained shear strengths, B & C, are necessary, but not sufficient, to evaluate the residual deformation by:

a) slip displacement analysis by Newmark-D method; and

b) residual deformation analysis by pseudo-static non-linear FEM.

- **A**: drained peak stress ratio
- **B**: undrained peak stress ratio necessary to develop 15% shear strain
- **C**: undrained cyclic loading strength necessary to develop 15% double amplitude shear strain

Toyoura sand; $\sigma'_{vc} = \sigma''_{vc} = 98$ kPa
Undrained stress-strain behaviour of saturated soil to evaluate the residual displacement/deformation

- Residual deformation obtained by pseudo-static non-linear FEM not including slip displacement
- Slip displacement obtained by the Newmark method

Estimated total ultimate residual displacement/deformation:
1. Residual deformation not including slip displacement; plus
2. Slip displacement

Perfect-plastic behavior without degradation during seismic loading is assumed

Minimum safety factor by LE analysis, $F_s$ against Level 2 design seismic load

1. Residual deformation obtained by pseudo-static non-linear FEM not including slip displacement

Allowable value

Total residual displacement/deformation:
1. Residual deformation not including slip displacement; plus
2. Slip displacement
Undrained stress-strain behaviour of saturated soil to evaluate the residual displacement/deformation - 2

- **Initial ss ee ee**

Shear strain, $\gamma$

Undrained $\tau \sim \gamma$ relation

In pulse $n$

Initial

Continuous degradation by undrained CL

Apparent working shear stress, $\tau_w$

Actual $\tau_w$ (= soil shear strength, $\tau_i$) decreasing by cyclic undrained loading

Three consecutive zero-crossing points

Time history of apparent irregular working stress $\tau_w$ obtained by total stress seismic response analysis not taking into account both strength degradation by undrained CL and slip failure

"Increments of slip displacement" in all pulses where slip takes place (such as $s \rightarrow e$) are integrated to obtain the ultimate residual slip displacement

Actual $\tau \sim \gamma$ behavior of soil

Time

Shear strain, $\gamma$

Apparent working shear stress, $\tau_w$

Pulse $n$

$\tau_w$

$\tau_i$

0
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 1:

Time histories of stress, strain and slip

Laboratory stress-strain tests

Seismic load

Damage strain

Soil strength

Slip displacement

Damage strain = 1%, 2%, 5%...

Cyclic stress amplitude

Calculation of slip displacement by the Newmark-D method
Typical example of cyclic undrained triaxial tests on isotropically consolidated specimens compacted to \((D_c)_{1Ec}= 85 \%, 90 \% \text{ and } 95 \%\):

- Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 2:
Undrained stress-strain behaviour of saturated soil - Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 3:

- (a) Seismic load
- (b) Damage strain
- (c) Soil strength
- (d) Slip displacement
- (e) Damage strain vs. log(Nc)
- (f) Soil strength vs. Damage strain

Calculation of slip displacement by the Newmark-D method.
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 4:

Definition of a pulse

Buffer zone (i.e., zero-crossing is defined only by full crossing of this zone)
Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability

Relationship between SR and “$N_c$ necessary to develop a given damage strain*” obtained by a series of uniform cyclic undrained tests

Cumulative damage concept

Damage for damage strain* by pulse $i$: $D_i = (1/N_i)$

$SR_i = \frac{\tau_{cyc,i}}{\sigma_0'}$: cyclic stress ratio of pulse $i$

$\tau_{cyc,i}$: shear stress amplitude; and $\sigma_0'$: initial effective confining stress

Undrained stress-strain behaviour of saturated soil

- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 5:
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 6:

**Cumulative damage concept**

![Graph showing SR vs log(Nc)](image)

**Relationship between SR and “Nc necessary to develop a given damage strain** *” obtained by a series of uniform cyclic undrained tests

**Damage for damage strain** * by pulse i: \( D_i = \frac{1}{N_i} \)

**Total damage for damage strain** * caused by a series of irregular pulses until the end of pulse n:
\[
D = \sum_{i=1}^{n} D_i = \sum_{i=1}^{n} \frac{1}{N_i}
\]

If D becomes 1.0 at the end of pulse n, it is assumed that this damage strain * takes place in pulse n.
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 7:

\[ \text{SR} \]

\[ \text{SR}_i \]

\[ \log(\text{number of loading cyclic, } N_c) \]

**Cumulative damage concept**

Relationship between SR and “\( N_c \) necessary to develop a given damage strain**” obtained by a series of uniform cyclic undrained tests

Damage for damage strain* by pulse i: \( D_i = \frac{1}{N_i} \)

Then, we can find the damage strain at the end of pulse n at which the total damage \( D = \sum_{i=1}^{n} \frac{1}{N_i} \) becomes 1.0.
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 8:

By this procedure, “a given time history of irregular cyclic stresses causing a certain damage strain can be converted to “uniform cyclic stresses with an arbitrary combination of SR & N_c that develops the same damage strain”.

Damage for damage strain* by pulse i: \[ D_i = \frac{1}{N_i} \]

Relationship between SR and “N_c necessary to develop a given damage strain*” obtained by a series of uniform cyclic undrained tests

Cumulative damage concept
Uniform cyclic stresses equivalent to “irregular working stresses before the start of pulse n” obtained by the cumulative damage concept.

Equivalent $\tau \sim \gamma$ behavior after have been subjected to “equivalent uniform cyclic stresses obtained by the cumulative damage concept”
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 10:

Damage strain = 1%, 2%, 5%….

Calculation of slip displacement by the Newmark-D method
Undrained stress- strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 11:
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 12:

Initial undrained shear strength

Undrained shear strength after undrained CL, $\tau_{\text{damage}}$

Strain that has developed by undrained CL

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Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 13:

Typical example: loose Hokota sand

\[ \Delta u (kN/m^2) \]

\[ q_{\text{max}} (kN/m^2) \]

\[ q_{\text{max}} \]

\[ \rho_d = 1.44g/cm^3 (D_c = 85\%) \]

\[ \sigma'_{\text{mi}} = 100kN/m^2, \tau_d / \sigma'_{\text{mi}} = 0.112 \]

\[ N = 23, \varepsilon_{\text{damage}} = 10\% \]

Initial undrained ML: \( q, \Delta u \)

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Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 14:

Typical example: dense Hokota sand
Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:

1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 16:

Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:
1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.

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\*\* τ_d / σ'_mi = 0.19; N_c = 2
Undrained stress-strain behaviour of saturated soil
- Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 17:

Undrained shear strength after cyclic undrained loading becomes significantly smaller as sand becomes looser, due to:
1) lower initial undrained shear strength;
2) larger damage strain by undrained CL; and
3) a larger drop rate for the same damage strain.
Undrained stress-strain behaviour of saturated soil - Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 18:

Time histories of stress, strain and slip

Laboratory stress-strain tests

Seismic load

Damage strain

Soil strength

Slip displacement

Calculation of slip displacement by the Newmark-D method

Damage strain = 1%, 2%, 5%….

Cyclic stress amplitude

Time histories of stress, strain and slip

Laboratory stress-strain tests

Seismic load

Damage strain

Soil strength

Slip displacement

Calculation of slip displacement by the Newmark-D method

Damage strain = 1%, 2%, 5%….
Different soil models for different Newmark methods

- Newmark-S (with strain-softening: \( \tau_f = \tau_p \Rightarrow \tau_r \))
- Newmark-O (\( \tau_f = \text{drained residual strength} \ \tau_r, \text{fixed} \))
- Newmark-D (with degradation by undrained CL)
- Newmark-SD (with degradation by undrained CL & strain-softening)

These values of \( \delta \) & damage strain are different among different soil models.

- Large effects of compaction
- No effects of compaction
- Significant effects of compaction

0 \( \tau_i \)

Working stress, \( \tau_w \)

Time, t

Slip displacement \( \delta \)

Damage strain by undrained CL

Peak drained strength, \( \tau_p \)

Initial undrained strength, \( (\tau_i)_0 \)
Undrained stress-strain behaviour of saturated soil - Undrained strength during cyclic undrained loading for slip displacement analysis by Newmark-D method - 20:

Time histories of stress, strain and slip

Laboratory stress-strain tests

Damage strain = 1%, 2%, 5%...

Calculation of slip displacement by the Newmark-D method
Contour of soil shear strength

Slip displacement, $\delta = R \cdot \theta$
Slip displacement by Newmark-D analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake

Very large ultimate slip displacement:
- Slip continues after the moment of peak acceleration \((t = 97.01 \text{ s})\) due to continuing deterioration in the undrained shear strength.
Undrained stress-strain behaviour of saturated soil
- Undrained stress – strain relation in the course of cyclic undrained loading modelled for residual deformation analysis by pseudo-static non-linear FEM - 1:

Cyclic undrained torsional simple shear test on dense Toyoura sand applying ‘seismic’ random stresses at a constant shear strain rate

<table>
<thead>
<tr>
<th>Shear stress ratio, $\tau_{vh}/\sigma_{mc}'$</th>
<th>(a) $\tau_{vh}/\sigma_{mc}'_{max} = 0.8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strain, $\gamma_{vh}$ (%)</td>
<td>(b)</td>
</tr>
<tr>
<td>Excess pore water pressure ratio, $\Delta u/\sigma_{mc}'$</td>
<td>(c) $\sigma_{rc}' = 66$ kPa</td>
</tr>
<tr>
<td>Effective vertical stress ratio, $\sigma_{v}'/\sigma_{mc}'$</td>
<td>(d) $\sigma_{mc}' = 98$ kPa</td>
</tr>
</tbody>
</table>

1968 Tokachi-oki E.Q. main shock, Hachinohe (NS)

Stress-strain behaviour of compacted soils related to seismic earth-fill dam stability
Shear stress, $\tau_{vh}$ (x 98 kPa)

Effective vertical stress, $\sigma'_{v}$ (x 98 kPa)

Shear strain, $\gamma_{vh}$ (%)

Air-pluviated Toyoura sand (e= 0.671 Cyclic undrained simple shear)

Complicated $\tau_{vh} - \gamma_{vh}$ relation!
Complicated $\tau_{vh} - \gamma_{vh}$ relation, but smooth strain-hardening hysteretic $\frac{\tau_{vh}}{\sigma_v'} - \gamma_{vh}$ relation:

- Yielding starts when $\frac{\tau_{vh}}{\sigma_v'}$ exceeds the previous maximum value in each direction.
Undrained stress-strain behaviour of saturated soil
- Undrained stress – strain relation in the course of cyclic undrained loading modelled for residual deformation analysis by pseudo-static non-linear FEM - 1:

Cyclic undrained torsional simple shear test on **loose** Toyoura sand applying ‘seismic’ random stresses at a constant shear strain rate

Stress-strain behaviour of compacted soils related to seismic earth-fill dam
Air-pluviated Toyoura sand (e= 0.778)
Cyclic undrained simple shear

Complicated $\tau_{vh} - \gamma_{vh}$ relation!
Complicated $\tau_{vh} - \gamma_{vh}$ relation, but smooth strain-hardening hysteretic $\tau_{vh}/\sigma'_v - \gamma_{vh}$ relation:

- Yielding starts when $\tau_{vh}/\sigma'_v$ exceeds the previous maximum value in each direction.

- The $\gamma_{vh}$ value at the peak $\tau_{vh}/\sigma'_v$ state after having passed the yielding point (e.g. point h) can be determined only by the peak $\tau_{vh}/\sigma'_v$ value and the ML stress – strain relation starting from the origin (i.e., $o \rightarrow y1 \rightarrow F \rightarrow h$), not referring to previous cyclic loading histories.
The $\gamma_{vh}$ value at the peak $\tau_{vh}/\sigma_v'$ state after having passed the yielding point (e.g. point h) obtained by following the reloading $\tau_{vh} - \gamma_{vh}$ relation (e.g. $f'\rightarrow y2\rightarrow F\rightarrow h$) is the same as the value obtained by following the monotonic loading $\tau_{vh} - \gamma_{vh}$ relation starting from the origin (e.g. $o\rightarrow y1\rightarrow F\rightarrow h$) while not referring to previous cyclic loading histories.
The time history of the $\gamma_{vh}$ value at the peak $\tau_{vh}/\sigma'_{v}$ states after having passed the yielding point (e.g. the values at points f, h & j) can be obtained by following respective ML stress-strain relations starting from the origin, o, that have degraded by respective preceding cyclic undrained loading histories.

→ In a slope in which initial shear stresses are acting, most of the respective peak $\gamma_{vh}$ value remains as the residual value upon unloading.
The time history of the residual deformation of a slope may be obtained by a series of pseudo-static non-linear FEM analyses incorporating gravity and seismic loads while using respective ML stress – strain relations starting from the origin.

Approximately, the maximum value of this deformation can be considered as the ultimate residual deformation caused by a given seismic loading history.
A series of pseudo-static FEM analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake

The largest deformation (at $t=100.14$ s):
- not by the peak acceleration ($t=97.01$ s), but later after the stress-strain relation has deteriorated more.
A series of pseudo-static FEM analysis of old Fujinuma dam, which collapsed by the 2011 Great East Japan Earthquake

The largest deformation (at t= 100.14 s):
- not by the peak acceleration (t= 97.01 s), but later after the stress-strain relation has deteriorated more.
- This largest deformation is considered as the ultimate residual deformation.
Several important features of the drained & saturated-undrained stress-strain properties of soil in monotonic & cyclic loadings related to the seismic stability of earth-fill dam

Practical simplified seismic stability analysis needs appropriate balance among the methods chosen in the following items:

1) Criterion to evaluate of the stability:
   Global safety factor relative to a specified required minimum vs. Residual deformation relative to a specified allowable largest.

2) Design seismic load at a given site:
   Conventional design load vs. Likely largest load in the future

3) Stress – strain properties of soil:
   Actual complicated behavior vs. Simplified model

4) Relevant consideration of the effects of other engineering factors:
   - compacted dry density; soil type; etc.
Design seismic load at a given site:

Conventional design load:
- specified in many old seismic design codes
- defined as Level 1 design seismic load in new seismic design codes (introduced after the 1995 Great Kobe E.-Q.)

Likely largest seismic load during the lifetime of a given structure:
- defined as Level 2 design seismic motion in new seismic design codes (introduced after the 1995 Great Kobe E.-Q.)
Japanese Society for Civil Engineers (1996):

• **Level 1 design seismic motion:** It is a seismic motion with a high likelihood of occurring during the design lifetime of the concerned structure. It is required that, in principle, all new structures have sufficient seismic resistance to ensure "no damage" when subjected to this seismic motion.

• **Level 2 design seismic motion:** It is the strongest seismic motion thought likely to occur at the location of the concerned structure during its lifetime. It is required that the structure should not collapse, although damage that renders it unusable is acceptable if its functionality can be rapidly restored.

The relationship among “the design seismic load”, “design shear strength of soil” and “stability analysis method (i.e., global Fs vs. residual deformation)” is complicated due to historical reasons.
Actual behavior during severe earthquakes

- Well-compacted fill A:
  Examples:
  - high rock fill dams
  - modern highway embankments
  - modern railway embankments
  - earth-fill dams

- Poorly-compacted fill B:
  Examples:
  - old soil structures before introduction of modern design and construction codes and methods
  - residential embankments

The diagram shows a graph with the shear strength of fill, $\tau_f$, on the y-axis and working stress, $\tau_w$, on the x-axis. The dotted line represents the performance during severe earthquakes, with Level 1 on the left and Level 2 on the right. The area above the line indicates A (stable) and the area below the line indicates B (collapsed).
Typical conventional seismic design of soil structure

Design seismic load \((\tau_w)_d\): \(k_h = 0.15\) (Level 1 seismic coefficient)
Design shear strength \((\tau_f)_d^*\): Drained shear strength when \(D_c\) by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90 %)

→ Required min. \(F_s\) by limit equilibrium stability analysis = 1.2, for example

Well-compacted fill A:
The use of \(k_h = 0.15\) as Level 2 seismic load is on the unsafe side. However, the use of \((\tau_f)_d^*\) as the drained/undrained strength of well-compacted fill is on the safe side.

→ These two factors may be balanced.
Typical conventional seismic design of soil structure

- Design seismic load \( (\tau_w)_d \): \( k_h=0.15 \) (Level 1 seismic coefficient)
- Design shear strength \( (\tau_f)_d^* \): Drained shear strength when \( D_c \) by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90%)

\[ F_s = \frac{\text{Strength}}{\text{Load}} = 1 \]

Well-compacted fill A:
The use of \( k_h=0.15 \) as Level 2 seismic load is on the unsafe side. However, the use of \( (\tau_f)_d^* \) as the drained/undrained strength of well-compacted fill is on the safe side.

\[ \rightarrow \text{These two factors may be balanced.} \]

If only \( k_h \) is increased: i.e., if only \( (\tau_w)_d \) is increased \( \Rightarrow \) Under-estimate of stability by losing balance (i.e., collapse despite actual stable performance against level)

Solution:
1) use of level 2 design seismic load; and
2) use of realistic high soil strength \( (\tau_f)_d \) corresponding to actual \( D_c \) (> 90%) (using undrained strength when relevant)
Typical conventional seismic design of soil structure

Design seismic load \((\tau_w)_d\): \(k_h = 0.15\) (Level 1 seismic coefficient)
Design shear strength \((\tau_f)_d^*\): Drained shear strength when \(D_c\) by Standard Proctor (1Ec) is equal to the required minimum value (e.g., 90%)

\[ F_s = \frac{\text{Strength}}{\text{Load}} = 1 \]

Poorly-compacted fill B:
The use of \(k_h = 0.15\) as Level 2 seismic load is on the unsafe side.
Besides, the use of \((\tau_f)_d^*\) as “undrained strength of saturated poorly-compacted fill subjected to seismic load” is on the unsafe side.
→ These two factors are not balanced.

Only an increase in \(k_h\) is not sufficient, but use of realistic low \((\tau_w)_d\) is necessary to duly evaluate the stability against level 2 seismic load.

Solution:
1) Use of level 2 design seismic load and;
2) Use of realistic low soil strength \((\tau_f)_d^*\) if saturated/undrained during seismic loading.
CONCLUDING REMARKS - 1

Several important factors that influence the drained & saturated-undrained stress-strain properties of soil when subjected to monotonic & cyclic loading histories related to the seismic stability analysis of soil structures, including earth-fill dams, were discussed. The following are the main conclusions:

1) Compacted soils exhibit significant strain-softening associated with shear banding, resulting in progressive failure in a slope.
2) As the thickness of shear band increases with $D_{50}$, the rate of strain-softening becomes slower with $D_{50}$, so the failure tends to become less progressive, making the slope more stable.
3) Although the effect of dry density on the drained peak shear strength is large, the effect on the undrained shear strength is much more significant.
CONCLUDING REMARKS - 2

4) The drained strength of compacted soil exhibits strong inherent anisotropy in plane strain compression.
5) The strength obtained by different stress-strain tests (i.e., triaxial and plane strain compression and direct shear) could be largely different due to the effects of different angles between the $\sigma_1$ direction and the bedding plane direction; different ratios of $\sigma_2$ to $\sigma_1$ & $\sigma_3$; and different definition of friction angle.
6) For the limit equilibrium-based stability analysis of a slope under plane strain conditions, a practical simplified method that assumes the followings, yet takes into account the effects of compacted dry density and particle size, can be proposed:

   a) Isotropic stress-strain properties exhibiting strain-softening of which the rate decreases with an increase in $D_{50}$.

   b) Use of the peak & residual strengths determined by the conventional TC tests at $\delta = 90^\circ$.

   c) Use of the peak strength corresponding to somehow conservatively determined compacted dry density (i.e., slightly lower than the value that corresponds to the anticipated average of the actual values of $D_c$).

   d) Progressive failure is not taken into account.
CONCLUDING REMARKS - 4

7) Under undrained monotonic loading conditions, loose & dense saturated soils exhibit the shear strength that is significantly lower and higher than the respective drained shear strengths.

8) The undrained shear strength decreases by preceding cyclic undrained loading. The effects of dry density on the damaged undrained shear strength are significant due to the following trends with an increase in the dry density:
   a) the increase in the initial undrained shear strength;
   b) the decrease in the damage strain by preceding cyclic undrained loading; and
   c) the decrease in the degradation rate by damage strain.
9) For simplified stability analyses of a slope having saturated zones by “slip deformation by the Newmark method” and “residual deformation by the pseudo-static non-linear FEM”, the characteristic feature of undrained stress – strain properties described above can be modelled in a unified framework from the same results of a set of monotonic and cyclic loading undrained stress – strain tests of saturated soil.
THANK YOU FOR YOUR ATTENTIONS